# HUMAN ACTIVITY INDUCED VIBRATION IN SLENDER STRUCTURAL SYSTEMS


## ABSTRACT

Human activity induced vibrations in slender structural systems become apparent in many different excitation modes and consequent action effects that cause discomfort to occupants, crowd panic and damage to public infrastructure. Resulting loss of public confidence in safety of structures, economic losses, cost of retrofit and repairs can be significant. Advanced computational and visualisation techniques enable engineers and architects to evolve bold and innovative structural forms, very often without precedence. New composite and hybrid materials that are making their presence in structural systems lack historical evidence of satisfactory performance over anticipated design life. These structural systems are susceptible to multimodal and coupled excitation that are very complex and have inadequate design guidance in the present codes and good practice guides. Many incidents of amplified resonant response have been reported in buildings, footbridges, stadia and other crowded structures with adverse consequences. As a result attenuation of human induced vibration of innovative and slender structural systems very often requires special studies during the design process. Dynamic activities possess variable characteristics and thereby induce complex responses in structures that are sensitive to parametric variations. Rigorous analytical techniques are available for investigation of such complex actions and responses to produce acceptable performance in structural systems.

This paper presents an overview of existing code provisions for human induced vibration followed by studies on the performance of three contrasting structural systems that exhibit complex vibration. The dynamic responses of these systems under human induced vibrations have been carried out using experimentally validated computer simulation techniques. The outcomes of these studies will have engineering applications for safe and sustainable structures and a basis for developing design guidance,



## KEYWORDS:

Vibration, slender structures, multi-modal and coupled vibration, human induced loads, hybrid floor structure, composite floor, footbridge, design guidance.


## 1. INTRODUCTION

Recent advances in computational techniques, materials technology and visualisation aids for creation of complex geometric forms have resulted in structural systems where the dynamic excitation characteristics become significant in design. Examples are lightweight long span floor plates, footbridges and cantilever terraces of grandstands. The last couple of decades have seen an increased amount of emphasis for the attenuation of human induced vibrations by structural designers to prevent adverse effects such as occupant discomfort, panic and sometimes structural damage and deterioration. The most notable publicised structure that experienced alarming levels of adverse vibration on the opening day was the Millennium Footbridge in London, while several large cantilever stadium structures such as the Millennium Stadium, Cardiff are reported to have exhibited perceptible vibration response to crowd induced behaviour at musical events [1]. In Australia, steel deck composite floors in commercial buildings, airports and shopping centres have been reported to exhibit human induced vibration causing some concern to occupants. As a result structural systems designed in compliance with design codes and best practice guides that were available at the time have required post construction rectification and retrofit. These codes and guides are very often adequate for designing regular structural forms with reliable history of performance. However the luxury of such well-honed experience and knowledge is not available for slender or geometrically complex designs that use innovative structural systems and new materials. Vibration response of slender structural systems can be attributed to the nature of human activity, multi-modal dynamic response, resonance, damping and the material properties. The determination of the interactive effects of all these attributes requires rigorous analytical techniques supported by tests to validate performance and design accuracy. This paper provides an overview of current design codes and research work on three contrasting structural systems subjected to human induced vibration to illustrate the complexity of the problem. The three structural systems investigated are: (i) lightweight hybrid composite floor plate system, (ii) steel deck composite floor and (iii) footbridge with reverse profiled cables. Rigorous analytical methods validated with experiments have been used in the research. The research was motivated by the need to address the knowledge gaps in the complex behaviour of slender structures subjected to human induced vibration and the lack of comprehensive guidance in the design codes and good practice guides to address this complex problem.

## 2. OVERVIEW OF CURRENT CODE PROVISIONS FOR VIBRATION

### 2.1 General Design Codes

The guidance provided in the Australian Standards AS3600 [2], AS4100 [3], AS2327.1 [4], AS5100 [5], British Standards BS8110-1[6], BS5950 [7] and the Structural Euro Codes EN 1992, EN 1993 and EN1994 [8] covering concrete, steel, composite and bridge structures is generic and limited to isolating the vibration source, increasing the damping and limiting frequencies to control the effects of floor vibration induced by human activity.

### 2.2Australian Standard

The Australian Standard that relates directly to vibration is AS2670 [9]. It provides guidance on the evaluation of human exposure to whole-body vibration: Part 1 gives general requirements, while Part 2 treats continuous and shock induced vibration in buildings and presents base curves for acceleration limits.

### 2.3. ISO Codes

International Standardisation Organisation (ISO) Codes provide guidelines for occupancy comfort and operating criteria for structures are subjected to vibration. Currently there are three ISO publications: (i) general requirements for the evaluation of human exposure to whole-body vibrations in ISO 2631-1[10], (ii) evaluation of human exposure to vibrations in buildings (1-80Hz range) in ISO 2631-2 [11] and (iii) bases for the serviceability design of building structures and walkways subjected to vibrations in ISO 10137 [12]. ISO 2631-1[10] suggests the use of frequency-weighting functions to evaluate vibration for human perception/discomfort in both the vertical and horizontal directions. It describes the frequency weighting method and the method of determining the RMS acceleration. This ISO code also refers to the Vibration Dose Value (VDV), which can be considered as the cumulative measurement of the vibration level received over a period of time, and explains the method of determining VDV. ISO 2631-2 [11] has extended requirements compared to ISO 2631-1 [10], but it does not provide any guidance on vibration assessment based on acceptable limits.

The latest edition, ISO 10137 [12] provides base curves with acceptable RMS acceleration limits for vibration assessment. As the acceptable vibration level varies with the frequency of the motion, the acceleration needs to be frequency weighted. If the ratio of peak to RMS value of weighted acceleration is $> 6$, use of VDV is suggested as the acceptance criteria for residential building floors. This ISO code also provides damping ratios for different types of floor structures and walkways. It suggests the use of the simplified methods in the practice guides (discussed below) or numerical techniques such as the finite element and boundary element methods for determining the vibration response and acceleration of structures.

## 2.4 BS Codes

Currently there are two relevant British Standards. BS 6841 [13] provides general requirements for the measurement and evaluation of human exposure to whole-body vibration and repeated shocks. BS 6472-1 [14] gives guidance on the evaluation of human exposure to building vibrations (1- 80-Hz range). It does not have the base curves and associated multiplying factors given in the previous BS publication [13], but it recommends VDV as the only method to evaluate vibration. Acceptable VDV limits for various occupancy classes are given.

BS 6472-1 [14] refers to existing methods to calculate vibration response of simple structures such as rectangular plates under harmonic or impulsive loads. It suggests the use of finite element techniques for other floor structures. Excitation functions for human activities are provided in this standard which also highlights the use of realistic damping based on previous experience with similar floor structures. This standard requires the acceleration to be frequency-weighted using the charts provided. VDV can then be calculated using the formulae in the standard and used to determine the acceptability of the vibration by referring to the base curves in [13].

## 2.5 Practice Guides

The American Institute of Steel Construction (*AISC) Design Guide 11* [15] and the Commentary D of the National Building Code of Canada [16] are commonly used in North America. They use the peak unweighted accelerations as the acceptability criteria for vibration control in building floors for different occupancy types. These limits are based on the recommendations made by Allen and Murray [17] in a previous publication, and do not consider the influence of vibration duration and frequency on the acceptable limits. AISC Design Guide 11 [15] provides a method to determine the fundamental frequency and peak acceleration of concrete/steel framed floor structures which are then used to check compliance. Walking and rhythmic activities are used in the analysis. These simple methods cannot be directly adapted to other types of floors.

Design of Floors for Vibration: A New Approach by the Steel Construction Institute (SCI) of the U.K. [18] is a more recent practice guide that can be used in the design of floor structures for vibration. This publication replaced the previous one by Wyatt [19] and uses frequency-weighting functions and acceptable VDV limits presented in the BS publications [13] and [14] respectively to assess the vibration due to human movements. This practice guide uses the empirical equations developed by Ellis [20] to calculate the VDV from peak weighted acceleration. A lower limit of 3 Hz is recommended for the frequency of floor structures.

This SCI practice guide [18] also provides methods of analysis, design and assessment of vibrations in steel framed concrete composite floors and floors using C and Z shaped steel joists with screw-fixed floor boarding. Finite element modelling is suggested for other types of structures. Acceleration response must be frequency weighted and used to calculate the RMS acceleration. The response factor, which is the ratio of the RMS acceleration to the base value given in BS 6472-1 [14], is then calculated and used as an assessment criterion. If this response factor is higher than the acceptable limits, VDV must be calculated and checked against the acceptable limits.

## 2.6 Grandstands

The people utilising grandstands and stadiums often jump or move in unison in response to sports, concerts or similar events. These structures therefore need provisions different to those in codes which apply to vibrations in buildings. ISO10137 [12] applies to structures that are exposed to rhythmic movements such as those imposed on a structure during sports events, concerts or any other activity with a large audience. It specifies criteria for the (i) comfort of passive persons (non participants) and (ii) overall safety of the audience and the structure.

Dynamic performance requirements for permanent grandstands subject to crowd action [21] pertain to grandstand floor structures with spans >6m or cantilevers >2.5m. There are frequency limits in lieu of dynamic evaluations when vertical and horizontal natural frequencies are less than 6 Hz and 3 Hz respectively. If the vertical natural frequency in any seating deck is < 3.5Hz, crowd management strategies are required. Only static load limits are provided for sway motions

### 2.7 Footbridges

ISO 10137 [12] is applicable to both floor structures and footbridges and presents a procedure for determining vibration levels of the receiver. It also provides guidance for analysing footbridges to ensure that the vertical accelerations do not exceed the specified limits. This standard refers to the practice guides [15, 19] and an IABSE publication [22] for further details. It alerts the possibility of coupled torsional vibration in footbridges and the importance of horizontal vibration when the natural frequency is less than 1.3 Hz.

The official commentary of the Canadian Highway Bridge Design Code CAN/CSA-S6-06 [23] presents a simplified method for calculating the acceleration of bridges and appropriate acceleration limits based on the first flexural frequency of the bridge. This simplified method is applicable only to one, two or three span pedestrian bridges that act as beams. This standard requires special design consideration when natural frequencies are less than approximately 4 Hz and suggests detailed dynamic analysis to obtain the acceleration response if the footbridge is used extensively by joggers as well as walkers.

According to BS 5400- Steel, Concrete & Composite Bridges Part 2 [24], vibration serviceability criteria for a footbridge are met if the fundamental natural frequency exceeds 5 Hz and 1.5 Hz in the vertical and horizontal directions respectively. If not, it imposes a limit for the maximum vertical acceleration of the superstructure and provides a procedure for its calculation for symmetric and simply supported bridges.

Eurocode5-Design of Timber Structures [25] provides some information on how to analyse bridge structures under human loads and the design values for the vertical acceleration of the structure for a few different load cases. It also provides some design criteria for sway vibration in footbridges. This code requires:(i) footbridges under crowd loads to have lateral frequencies outside the range 0.5 to 1.3 Hz, or a strategy for mitigating resonant sway vibration to be developed at design stage and (ii) accelerations to be limited to 0.7 and 0.2 $m/s^2$ in the vertical and lateral directions respectively.

### 2.8 Summary

Vibration limits in terms of accelerations (peak, RMS) or VDVs are presented in codes and practice guides. The practice guides provide simplified methods to determine vibration response, but these are limited to certain types of structures. Both the codes and practice guides suggest the use of Finite Element (FE) techniques to determine the vibration response of structures, but they do not provide adequate guidance on the appropriate FE techniques for different types of floor structures. FE techniques that cover coupled and multi-modal vibration are required for slender footbridges and for slender composite floor structures under pattern loading. None of the codes and practice guides that were reviewed provided appropriate and adequate guidelines for evaluating the complex response resulting from coupled and multimodal action effects. The simplified procedures for the design of building floors for human induced vibration in the 2009 JCR report [26] are not applicable to structures which exhibit complex vibration.

## 3. HYBRID FLOOR STRUCTURE

### 3.1 Introduction

Research is underway to develop an innovative hybrid-composite floor plate system (HCFPS) using polyurethane (PU), glass-fibre reinforced cement (GRC) and thin perforated steel laminate, as shown in

Figure 1. HCFPS is configured such that the positive inherent properties of individual component materials are combined to offset any weakness and achieve the optimum performance. Width of the HCFPS is normally limited to suit prefabrication and transportation requirements. Length of the HCFPS can be varied by changing the material properties, component material thickness and beam depth. In light weight structures such as the HCFPS it is important to satisfy the serviceability limit states for overall performance.

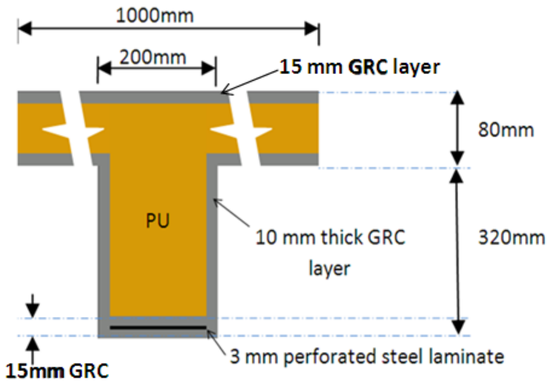



Figure 1: Cross section of HCFPS

### 3.2 Static Performance

Static performance of a 6m long and 4m wide HCFPS (consisting 4 of the sections shown in Figure 1), simply supported at its ends and subjected to dead and imposed loads of 1kpa and 3kpa respectively was investigated using FE techniques. This hybrid structure was modelled using the FE program ABAQUS 6.9-1 with C3D8R eight node liner brick elements having maximum element size of 20mm. Reduced integration and hourglass control were implemented along with full fixity between layers. Material properties were obtained by testing individual components and those required for the FE model are listed in Table 1. Stresses and deflection at the mid span section were determined. Results showed that stresses were well within the ultimate capacities of the component materials - GRC: 3MPa in tension and 8.1 MPa in compression, PU: 0.02 MPa in both tension and compression and Steel: 145MPa in tension. The mid span deflection was less than the span/360 limit under service loads. These results indicate that the static performance of the HCFPS is acceptable. Details are presented in [27].

### 3.3 Dynamic Performance

The dynamic performance of the HCFPS (or any other) floor structure under human-induced vibration is (normally) evaluated against acceptance criteria in terms of frequency and acceleration limits. The codes and practice guides reviewed earlier do not provide methods of finding such limits for newer floor structures such as HCFPS. The dynamic performance of the HCFPS was therefore investigated using experimentally validated FE techniques.

### 3.3.1 Experimental Testing

Heel impact tests were carried out on three panels to determine the damping coefficient and first natural frequency. Figure 2 shows a typical test panel. Each panel was 3.2m long and 1m wide and simply supported at the ends. The acceleration response for the heal impact was measured. Using the acceleration-time plots the average damping coefficient was calculated as 5.1% based on the logarithmic decrement method. The reasonably high value of damping for this low mass structure is due to its energy absorbing PU core. The average first natural frequency was obtained as 22.8Hz from the acceleration response using FFT analysis.

An FE model of the HCFPS was developed as described earlier. Free vibration analysis of the FE model was carried out to obtain the first natural frequency as 23.64. This value agrees reasonably well with the average value of 22.8 obtained through experimental testing and indicates that the fundamental frequency is well predicted by the FE model. This value of the fundamental frequency is greater than the minimum values suggested in the codes and practice guides. However, the peak acceleration of the HCFPS obtained from experimental testing of the bare floor was high compared with the acceptance limit given in [15]. A parametric study was therefore carried out by improving the material properies.

Figure 2: Test panel

| Properties | Polyurethane | GRC | Steel |
|---|---|---|---|
| Density (kg/m$^3$) | 99.8 | 1983 | 7800 |
| Young modiulus (Mpa) | 22.4 | 4998 | 210,000 |
| Poisons ratio | 0.3 | 0.15 | 0.3 |

Table 1 Material properties for FE models

### 3.3.2 Dynamic performance HFPS

A simply supported HCFPS 6m long 4m wide with improved material properties shown in Table 2 was used to investigate the dynamic performance under human induced loads, using FE techniques. Mesh used in the FE model is depicted in Figure 3 where maximum element size is 20mm. The natural frequencies of this panel were determined as 32.8Hz, 34.3 and 40.1 for the fundamental, second and third modes respectively. The fundamental mode is shown below and the fundamental frequency is well above the minimum limits given in codes and practice guides.

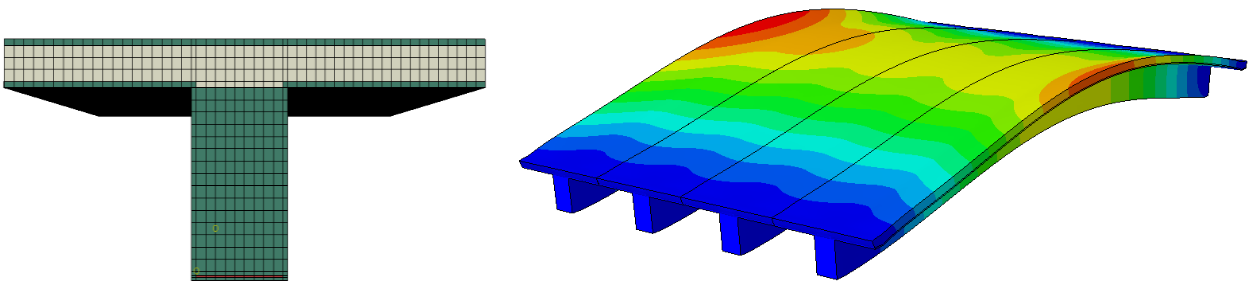


Figure 3: Mesh pattern of the FE model and fundamental mode

| Properties | Polyurethane | GRC | Steel |
|---|---|---|---|
| Density (kg/m$^3$) | 300 | 1800 | 7800 |
| Young modiulus (Mpa) | 151.6 | 18,000 | 210,000 |
| Poisons ratio | 0.3 | 0.15 | 0.3 |

Table 2 Material properties for FE models

### 3.3.3 Acceleration response under walking loads

This panel was analysed under human induced walking loads to obtain the acceleration response. A mathematical model for the human induced load *F(t)* [12] as given in Equation (1) was used to excite the FE model.

$$F(t) = Q\left[1 + \sum_{n=1}^{k} \alpha_n \sin\left(2\pi n f_p t\right)\right] \tag{1}$$

In the above equation $Q$ is the static weight of the person, $\alpha_n$ the Fourier Coefficients, $f_p$ the pacing frequency, $t$ the time and $n = 1\ldots k$. The numerical values of the first there Fourier coefficients used to model the human-activities are given in Table 3. The static load was ramped for the first second and then the dynamic walking load was applied.

| | Q ( kN/m2) | $f_p$ (Hz) | $\alpha_1$ | $\alpha_2$ | $\alpha_3$ |
|---|---|---|---|---|---|
| Walking | 0.75 | 2.2 | 0.4 | 0.1 | 0.06 |

Table 3 Numerical values for the load model [12]

Acceleration response of the HCFPS was obtained for 5% damping as this was the value obtained from the experimental studies with a bare floor. Higher damping could arise in a floor with partitions and finishes. An initial peak acceleration of about 1% g was observed, as seen in Figure 4, but the acceleration dissipates with time due to the improved material properties and 5% damping ratio.

The value of this initial acceleration was compared with the limits given in the AISC Steel Design Guide 11[15] where the peak acceleration limit for residential and office floors with the fundamental frequency of 31 Hz is about 1%g. The peak acceleration of this HCFPS therefore does not exceed the prescribed limit. This work demonstrates how rigorous analysis techniques with experimental validation can be used to control performance criteria for human induced vibration effects.

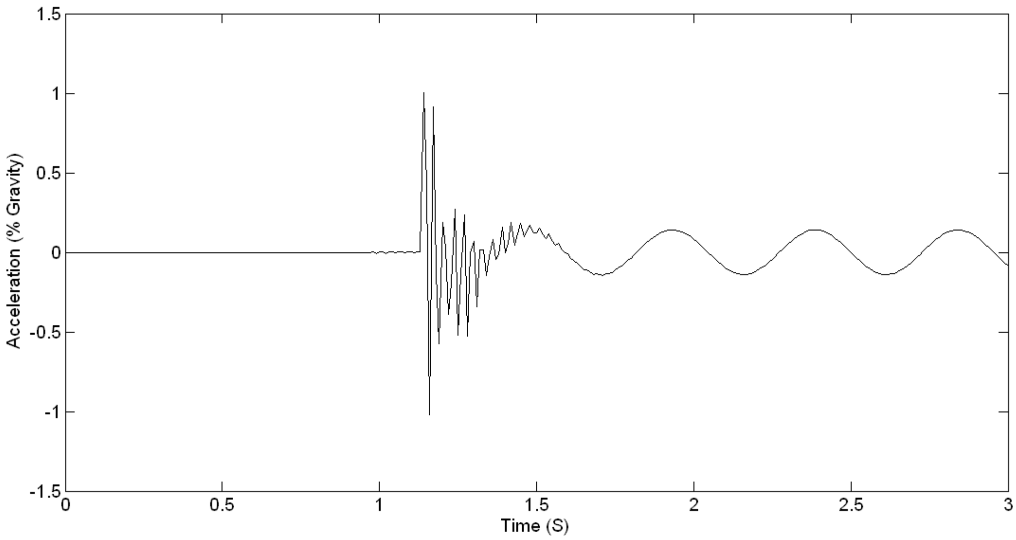



Figure 4: Acceleration response of HCFPS

### 3.4 Discussion

Research has shown that the ultra-light HCFPS has the potential to be used as a floor plate system in building structures provided its vibration performance meets acceptable criteria for occupant comfort and perception. Due to low mass, its peak acceleration under human induced loads is comparatively higher than that of conventional floor structures. However, the studies demonstrate how peak acceleration of

HCFPS can be controlled within prescribed limits. Design codes and practice guides do not provide adequate guidelines for investigating and designing similar floor plate systems for human induced vibrations. This research demonstrates how rigorous analytical methods using FE techniques validated by experiments can be effectively used for performance evaluation of lightweight floor plate systems made of non-traditional materials.

## 4. STEEL DECK-COMPOSITE FLOOR

Steel-deck composite floor systems use high strength materials to achieve longer one way spans. These types of floors, especially those with multi-occupancies, have experienced vibration problems under human induced loads. This paper treats the dynamic response of such a floor structure under human-induced loads using Finite Element (FE) techniques, which have been validated through experimental testing for free vibration, static and dynamic responses [28].

### 4.1 Dynamic analysis of multi-panel floor model

Multi-panel floor systems with four and nine panels were investigated, but this paper will present results of only the four panel systems [28, 29]. A four panel floor system with a dovetailed profile and total area of 16mx15.6m, having column grids of 8.0m x 7.8m, is considered. The columns and the primary beams along the panel edges in the spanning direction were 530 UB 82. The transverse beams were 360 UB 45 and simply supported on the primary beams. The slab had 150mm thick concrete cast in situ over 1 mm dovetail profiled steel-deck. The Young's modulus and Poisson ratio were 31GPA and 0.2 for the concrete and 205GPa and 0.3 for the steel respectively. Further details are given in [28]. The concrete slab is modelled using 3D 6 node hexagonal solid elements and the steel-deck modelled using S4R5 quadrilateral shell elements while the supporting beams and columns are modelled with beam elements. The surface mesh size was 25 x 32 mm while the solid mesh was 25 x 32 x 50 mm. Four patterns of half sinusoidal load functions representing dance type human activity (more onerous than walking or running) were developed and applied with load intensity, foot contact ratio and frequency as variable parameters. These loads can be defined by *F(t)* in Equation (2) in which $Q$ is the human load density, $t_p$ the contact duration, $T_p$ the duration of the cyclic loading and $\alpha = t_p/T_p$ is foot contact ratio [28].

$$F(t) = (\pi Q/2\alpha)\, \mathrm{Sin}\pi t/t_p),\ 0 \le t \le t_p \quad \text{and} \quad F(t) = 0,\ \ t_p \le t \le T_p \tag{2}$$

Four values of foot contact ratios $\alpha =$ 0.25, 0.33, 0.50 and 0.67 pertaining to high impact jumping, normal jumping, high-impact aerobics and low impact aerobics respectively are used with two different load densities of $Q$ = 0.2 kPa and $Q$ = 0.4 kPa and a dead load of 3.5 kPa. Damping levels of 1.6%, 3%, 6% and 12% and activity frequencies in the range 1.5Hz to 3.5 Hz are considered. Pattern loads PL1 acting on a single panel and PL2, PL3 and PL4 acting on 2 adjacent panels parallel and perpendicular to the span, and diagonal spans respectively are applied one at a time, to capture the dynamic response [28].

### 4.2 Results and discussion

Figure 5 shows the shapes of the first 4 modes of vibration and it is evident that the pattern loads could excite all these modes.

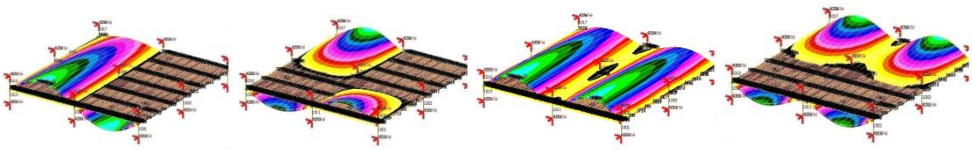


Figure 5: First four mode shapes with frequencies 4.0, 5.4, 5.9 and 6.9 Hz, L to R

Maximum values of the accelerations for each particular load are plotted in perceptibility diagrams for different operating conditions and occupancies. Figure 6 shows a typical diagram under PL2 loading for $\alpha = 0.33$ (normal jumping activity) and human density Q = 0.2 kPa. This event requires damping levels of 6.0% or more in the activity panels to avoid human discomfort, and 12.0% or more damping for occupancy 2 in the non-activity panels. Occupancies 1, 2, 3 and 4 are gymnasiums, shopping malls/warehouses, offices/residences and hospitals/laboratories respectively. The perceptibility scales for high impact aerobics ($\alpha = 0.50$) under PL2 showed that the activity panel requires 3% or more damping. The non-activity panels were suitable for occupancy 2 at 6.0% or more damping. Similar graphs were obtained for the other load cases [28].

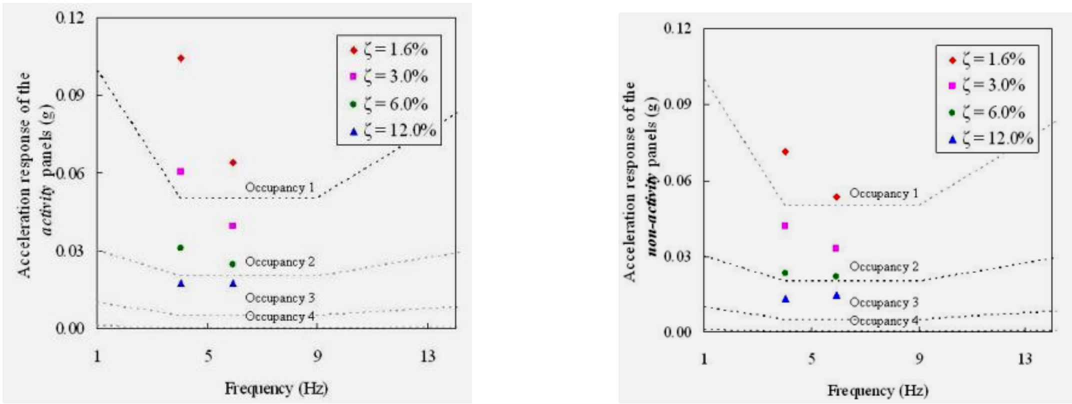



Figure 6: Perceptibility diagrams under PL2 loading.

Depending on the pattern loading, foot contact ratios and damping levels, the acceleration responses reported maximum values at activity frequencies of 2.0 Hz, 2.7 Hz and 2.95 Hz. The Fourier Amplitude response spectrums for the acceleration of the structural system under PL1 at damping level 1.6% and contact ratio $\alpha = 0.25$ are shown in Figure 7. In the left hand Figure there are two distinct peaks at frequencies of 4.0 and 6.0 Hz corresponding to the excitation of the 1st and 3rd modes of the floor system by the 2nd and 3rd harmonics of the activity frequency of 2Hz. The right hand Figure depicts a single peak near 5.9 Hz corresponding to the excitation of the 3rd mode by the 2$^{nd}$ harmonic of the activity frequency of 2.95Hz. Analogous results were obtained for PL2 at the same activity frequencies of 2 and 2.95Hz [28, 29]. Fourier Amplitude Spectrums for the acceleration under PL3 and PL4 loadings provided further evidence of excitation of the second and third modes by the higher harmonics of the activity frequencies.

From the above results it is evident that in addition to the fundamental mode, higher modes of vibration can be excited in steel-deck composite floors by higher harmonics of the forcing dynamic activity, resulting in multi modal and possibly coupled vibration. Analogous results were obtained with the nine panel floor system [29]. Current simplified methods in codes and practice guides for assessing floor vibration are primarily based on the fundamental natural frequency and confined to the activity panel. The analytical techniques used in this research can be effectively applied to treat footfall induced vibration in slender steel deck composite floor plate systems.

.

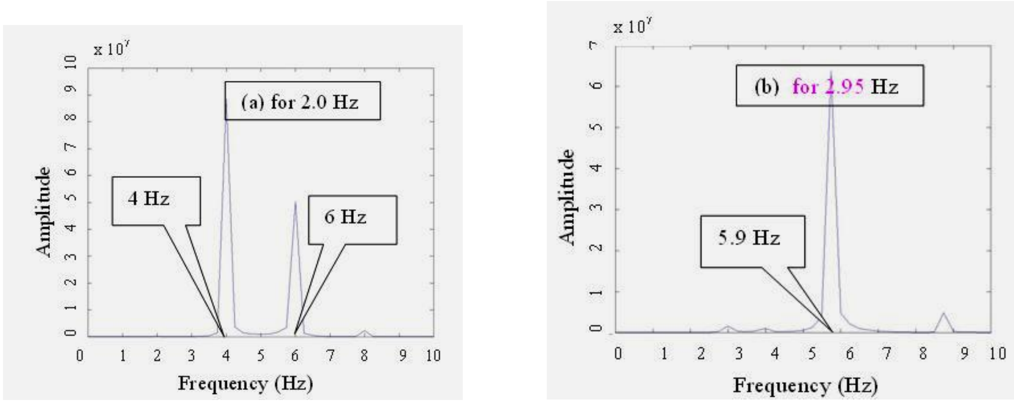



Figure 7: Response spectrum under PL1 loading at 2 Hz and 2.95 Hz

## 5. CABLE SUPPORTED FOOTBRIDGE

### 5.1 Bridge model

To study the complex vibration in slender footbridges, a cable supported bridge with top, bottom and side cables is considered for FE analysis (Figure 8). The bridge was modelled using 3D beam/frame elements. The second moment of areas of the elements used to model the cables were suitably adjusted to simulate cable action and 20 elements were used to model one segment between two adjacent bridge frames. The bridge deck units are simply supported on the beams. The FE model was validated through experimental testing for free vibration and static responses. Structural and material details of the model and the validation are given in [30]. The top cables support the gravity loads and the internal forces induced by the bottom pre-tensioned cables which have reverse profiles and introduce extra internal vertical forces to transverse bridge frames and the top cables. The side cables are a pair of bi-concave cables in the horizontal plane and provide extra horizontal stiffness. In some footbridges the side cables are omitted. The cable system is able to mobilise stiffness components in all the directions and capture multimodal and coupled behaviour. There are other studies on footbridges reported in the literature, for example Zivanovic et al [31].

### 5.2 Vibration Modes and coupling of modes

Results from free vibration analysis show that this footbridge with shallow cable profiles, exhibits 2 types of coupled modes: coupled lateral-torsional (LmTn) and coupled torsional-lateral (TmLn) modes, where L and T refer to lateral and torsional modes and m and n are the number of half waves [32]. LmTn modes are dominated by lateral vibration, while TmLn modes are dominated by torsional vibration. Vertical vibration modes are mostly un-coupled while the longitudinal modes disappear from the first twenty when the bottom reverse profiled cables are tensioned. It is also evident that mode coupling defined by $U_v/U_l$ in Figure 9, is affected by cable sag, cable section as well as bridge span and influence the LmTm and the TmLm modes differently [32].


Figure 8: Cable supported footbridge: (l) isometric view and (r) x-section

### 5.3 Dynamic response under human-induced loads

Synchronous excitation can be caused by the combination of high density of pedestrians and low natural frequencies of bridges within the range of pacing rate [33]. When synchronization occurs, footbridges resonate and a part of the pedestrians will change their footfalls to match the vibration. To model the synchronous crowd walking dynamic loads, the following assumptions are made: (1) 20% of pedestrians participate in the synchronization process and generate vertical and lateral dynamic loads, while the remaining 80% pedestrians generate only static vertical load on the bridge deck as they walk with random pacing rates and phases. (2) The force generated by a footfall has components in the vertical, lateral and longitudinal directions. The vertical and lateral components follow Wheeler's force-time functions and the magnitude of the lateral component is (4%) of the vertical component. The longitudinal component is not important for the lateral vibration and is neglected [33].

Walking dynamic load will therefore have 3 parts: vertical dynamic force, lateral dynamic force and vertical static force. To simulate different load patterns, walking dynamic load can be modelled as uniformly distributed or eccentrically placed along one side of the bridge. Figure 10 shows the vertical load function $F_n[t]$ of one footfall under normal walk in which $T_n$ and $T_{nc}$ are the period and foot contact time respectively. There are 4 types of walking activities: slow walk (< 1.8Hz), normal walk (1.8Hz ~ 2.2Hz), brisk walk (2.2-2.7Hz) and fast walk (greater than 2.7Hz). When the pacing rate of synchronized pedestrians coincides with a natural frequency, the bridge is subjected to synchronous excitation and vibrates in resonance. The vibrating mode depends on the distribution of dynamic loads and natural frequencies. The first coupled lateral-torsional mode ($L_1T_1$) and first vertical mode ($V_1$) are easier to be excited by crowd loads than other vibration modes when the entire deck is full of pedestrians, as these 2 modes are one half wave symmetric vibration modes. The first coupled torsional-lateral mode ($T_1L_1$) or torsional mode ($T_1$) are not easy to be activated when the bridge is uniformly and fully loaded, but can be excited by eccentrically distributed walking dynamic loads [33].

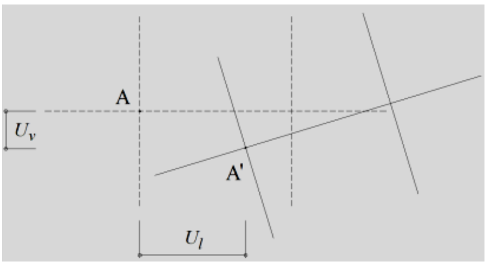



Figure 9: Deformed bridge frame

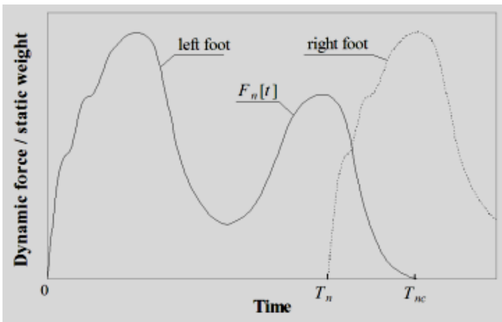
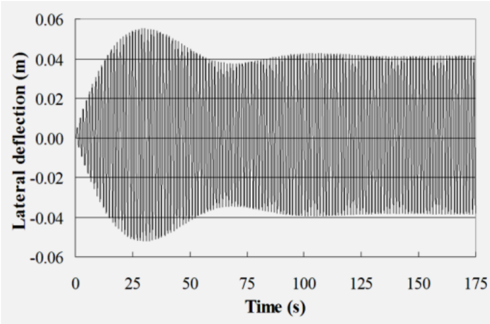



Figure: 10 Normal walking load

Figure 11 shows the dynamic response of a bridge excited by a uniformly distributed crowd at a pacing rate of 1.5Hz. This footbridge has top and bottom cables and a fundamental lateral frequency of 0.75Hz. When pedestrians walk cross the footbridge with the pacing rate of 1.5 Hz, the first coupled lateral-torsional mode $L_1T_1$ is excited at the frequency of 0.75 Hz which is half of the pacing rate, and resonant vibration in the lateral direction is expected. It can be seen that the amplitude of the lateral vibration increases to the maximum value and then fluctuates and before becoming steady. In the vertical direction, there is a deflection about which the bridge vibrates with amplitude that is much smaller than the lateral one. This vertical vibration also tends to be steady after initial fluctuations. The vertical vibration is contributed by three loads: static load, vertical dynamic load and the lateral sway of bridge frame under lateral dynamic load, with the static load being most dominant. It is evident that the bridge does not resonate in the vertical direction when subjected to the vertical dynamic load and the maximum vertical dynamic deflection is mainly produced by the resonant lateral sway. This behaviour was independent of cable configurations or cable sizes.



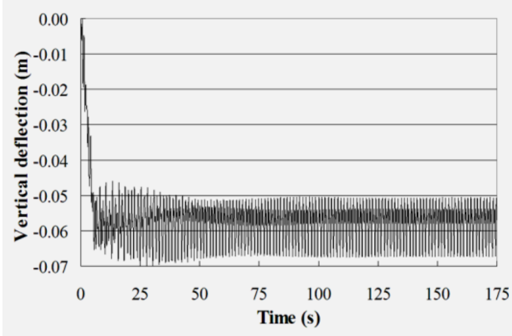



Figure 11: Lateral (l) and vertical (r) deflection response at pacing rate of 1.5 Hz.

Bridge behaviour with different cable configuration and sizes and with different fundamental frequencies was investigated. It was found that large amplitude lateral vibration is mainly caused by resonant vibration under coupled vibration modes. When $L_1T_1$ is excited, large amplitude lateral vibration is induced by the lateral dynamic force; while when the $T_1L_1$ mode is excited, there is excessive lateral vibration with increasing mean value caused by the vertical dynamic force and enhanced by the lateral dynamic force. When the first vertical mode is excited, the amplitude of lateral vibration is quite small as it is mainly caused by the lateral dynamic force. These research findings provide new knowledge on the vibration characteristics of slender footbridges.

## 6. CONCLUSIONS

The main findings from this study are:

- The performance of the structures treated herein clearly emphasise the complexity of vibration problems resulting from parametric variations that simplistic methods in the current codes and good practice guides fail to address.
- Investigations of human activity induced floor and bridge deck vibrations based on existing codes and practice guides are limited to either the first mode of vibration or a few uncoupled modes in the vertical direction of a single panel directly exposed to the activity.
- In slender multi-panel floor structures and bridge decks multi-modal and coupled vibrations under patterned load effects are significant. In addition higher modes have an impact on adjacent floor panels not exposed to direct activity.
- The performance of the multi-panel floor structure illustrates the complexity of asymmetry, but would show up as acceptable using the code and design guide prescribed methods.
- The hybrid composite floor plate system has high frequencies and adequate damping due to its low mass and the energy absorbing intermediate layer respectively. The present codes and practice guides do not have guidance for determining the frequency and acceleration of these new types of structures.
- Coupled excitation in the footbridge shows the need for such evaluations for which there are no guide lines and the current knowledge is shrouded in mystery and intrigue although the problems have been reported over decades.
- The simple methods provided in the current codes may be adequate to identify potential problems in floor plates and bridge decks, but the true response in relation to multimodal and coupled behaviour needs to be investigated with dynamic evaluations similar to those described in this paper.
- Load models based on the characteristics of the human activity can be developed and applied to the slender structure. The maximum response can be compared with the established perceptibility/comfort criteria to evaluate the serviceability of the structures.

- The same approach is applicable to grandstands, where occupant safety, panic stricken crowd behaviour and significant vertical deflections coupled with sway modes are more critical than comfort criteria.
- The research findings in this paper are intended to enhance the understanding on the complexity of vibration in slender structures and contribute towards the development of future design guides.

## 7. REFERENCES


[1] Structural Safety 2000-01, 13th report of Standing Committee On Structural Safety, May 2001, SCOSS, London, UK.

[2] AS3600, Standards Australia 2009. *Concrete structures*, Sydney, NSW, Australia.

[3] AS4100 Supplement 1, 2007. Standards Australia. *Steel structures commentary* (supplement to AS4100-1998), Sydney, NSW, Australia

[4] AS2327.1, Standard Australia 2003. *Composite structures. Part 1: Simply supported beams*, Sydney, NSW, Australia

[5] AS 5100.2, Supplement 1.2007. Standards Australia, *Bridge design-Design loads commentary* (supplement to AS 5100.2, 2004), Sydney, NSW, Australia.

[6] BS 8110-1 British Standards.1997. Part 1: *Code of practice for design and construction*, BSI, UK

[7] BS 5950 British Standards. 1994. *Structural use of steelwork in building*. Part 4: Code of practice for design of composite slabs with profiled steel sheeting. BSI, UK.

[8] Structural Euro Codes: EN 1992 *Design of concrete structures*, EN 1993 *Design of steel structures* and EN 1994 *Design of composite steel and concrete structures*, BSI, UK.

[9] AS2670, *Evaluation of human exposure to whole-body vibration*. Part 1(2001): *General requirements* and Part 2 (1990) *Continuous and shock-induced vibration in buildings (1-80Hz)*, Standards Australia, NSW, Australia.

[10] ISO 2631-1. 1997. *Mechanical vibration and shock Evaluation of human exposure to whole-body Vibration-Part 1: General requirements*, International Standardisation Organisation, Geneva, Switzerland.

[11] ISO 2631-2. 2003. *Evaluation of human exposure to whole-body vibration- Part 2: Vibration in buildings (1 to 80 Hz)*, International Standardisation Organisation, Geneva, Switzerland.

[12] ISO 10137. 2007. *Bases for Design of Structures- Serviceability of Buildings Against Vibrations,* International Standardisation Organisation, Geneva, Switzerland.

[13] BS 6841 British Standards. 1987. *Measurement and evaluation of human exposure to whole-body mechanical vibration and repeated shock*, BSI, UK

[14] BS 6472-1 British Standards. 2008. *Guide to evaluation of human exposure to vibration in buildings—Part 1: Vibration sources other than blasting,* BSI, UK

[15] Murray, T.M., Allen, D.E. Ungar. E.E.(1997) AISC Steel Design Guide II: *Vibration due to Human Activity,* American Institute of Steel Construction, Inc. Chicago, USA

[16] NBCC .2005. *Users' guide-National Building Code of Canada: Structural commentaries - Commentary D: Deflection and vibration criteria for serviceability and fatigue limit states* Canadian Commission on Building and Fire Codes, National Research Council of Canada, Ottawa.

[17] Allen, D. E &. Murray, T. M. (1993) Design criteria for vibration due to walking, *AISC Engineering J.*, Vol 40, Issue 4, 117-129.

[18] Smith, A.L., Hicks, S.J., Devine, P.J. 2007. *Design of Floors for Vibration: A New Approach.* The Steel Construction Institute, Ascot, Berkshire, UK.

[19] Wyatt, T.A. 1989. *Design Guide on the Vibration of Floors*, The Steel Construction Institute, UK, Ascot, Berkshire, UK.

[20] Ellis, B. R., (2001) Serviceability evaluation of floor vibration induced by walking loads, *Struct. Eng.*, Vol 79, Issue 21, 30-36.

[21] *Dynamic Performance Requirements for Permanent Grandstands Subject to Crowd Action.* 2008. The Institution of Structural Engineers, London. UK.

[22] Bachmann, H., and Ammann, W.1987. *Vibrations in Structures: Induced by Man and Machines*, International Association of Bridge and Structural Engineering (IABSE),

[23] *CAN/CSA-S6-06. Canadian Highway Bridge Design Code*. 2006. Canadian Standards Association.
[24] *BS 5400-* Steel, *Concrete & Composite Bridges,* Part 2 2006. Appendix B, BSI, UK.
[25] *Eurocode 5*. 1997. *Design of Timber Structures-Part 2: Bridges, ENV 1995-2: 1997.*
[26] JCR Report 2009, Design of floor structures for human induced vibration, by M. Feldmann et al, Ed: G. Sedlacek, Ch. Heinemeyer and Ch Butz, European Commission on Joint Research Centre, EUR 24084 EN, Luxembourg.
[27] Abeysinghe, CM, Thambiratnam, DP, Perera, NJ (2011).*Development of Innovative Hybrid-composite floor plate system,* Proceedings, 13th International Conference on Civil, Structural and Environmental Engineering Computing, Chania, Greece.
[28] De Silva, SS, Thambiratnam, DP (2009). Dynamic characteristics of steel deck composite floors under human-induced loads, *Computers & Structures*, Vol 87, 1067- 1076.
[29] De Silva, SS, Thambiratnam, DP. 2011. Vibration characteristics of concrete-steel composite floor structures", *ACI Structural Journal*, Vol. 108, Issue 6, pp. 706- 714, November.
[30] Huang, MH (2006). Dynamic characteristics of slender suspension bridges, PhD thesis, QUT, 2006.
[31] Zivanovic, A, Pavic, A and Reynolds, P. (2007). Finite element modeling and updating of a Lively footbridge: the complete process, J. Sound & Vibration, Vol. 301, Iss 1-2, pp. 126-143
[32] Huang MH., Thambiratnam, DP, Perera, NJ. 2005. Vibration characteristics of shallow Suspension bridge with pre- tensioned cables, *Engineering Structures,* Vol 27. 1220-1223.
[33] Huang, MH, Thambiratnam, D.P. and Perera, N. (2008), "Coupling Coefficient and Lateral Vibration of Slender Suspension Footbridges", *Computers & Structures,* Vol. 86, pp. 27-37.